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Response of Wall Panels Subjected to Blast Loading

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RESPONSE OF WALL PANELS SUBJECTED TO BLAST LOADING

by

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INTRODUCTION

Since the development of nuclear weapons, voluminous information has been documented from weapons tests, bombing surveys, and theoretical work on the effects of these weapons on buildings and their structural elements. A review of much of this material reveals an almost complete lack of certain types of information which was needed to make refined structural damage predictions required by OCD for a variety of purposes, including:

- Casualty and injury predictions
- Debris predictions
- Predictions of blast effects on fire development and spread
- Prediction of damage to equipment and property in structures
- Guidance in construction design for protection from all weapon effects.

Some of the specific types of information that were lacking were element failure times, the amount of energy and/or impulse transmitted by a failing element to its supporting frame, and the effect of a variety of geometric considerations such as openings, support conditions and building orientation and size on the loading function. A long-range program of shock tunnel research on the loading, structural response, and debris characteristics of wall panels

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was begun in 1966 for the Office of Civil Defense under subcontracts from Stanford Research Institute in Menlo Park, California, which has been acting as the technical monitor for the program.*

The main emphasis to date has been directed toward the study of brittle materials, particularly wall panels constructed of non-reinforced brick. The rationale behind this choice of structural material was based on data collected during the National Fallout Shelter Survey which determined that non-reinforced brick accounted for the largest percentage of exterior walls (~ 38% of the total) and that brittle materials in general accounted for ~ 50% of all exterior partitions.

THE TEST FACILITY

The URS Shock Tunnel facility, used to conduct the test program, is part of a converted coastal defense complex. The shock tunnel, shown in Figs. 1 and 2, consists of a 163-ft long section of reinforced concrete.

The first 63 ft of the tunnel (the compression chamber) is 8 ft by 8.5 ft, lined with an 8 ft diameter by 3/8 in. thick steel cylindrical shell, held in place with rigid foamed-in-place urethene foam (2 lb/ft^3 density).

The remaining 100 ft of tunnel is used as the expansion chamber and has an 8.5 ft by 12 ft cross section capable of accommodating "full scale" wall panels (8 ft by 12 ft).

The tunnel is operated as a shock tube by means of the volume detonation technique, with primacord as the explosive material.[1] In this mode of operation, the primacord is distributed uniformly throughout a section (up to 63 ft)

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* Subcontract No. 11229(6300A-320) and 11618(6300A-250).

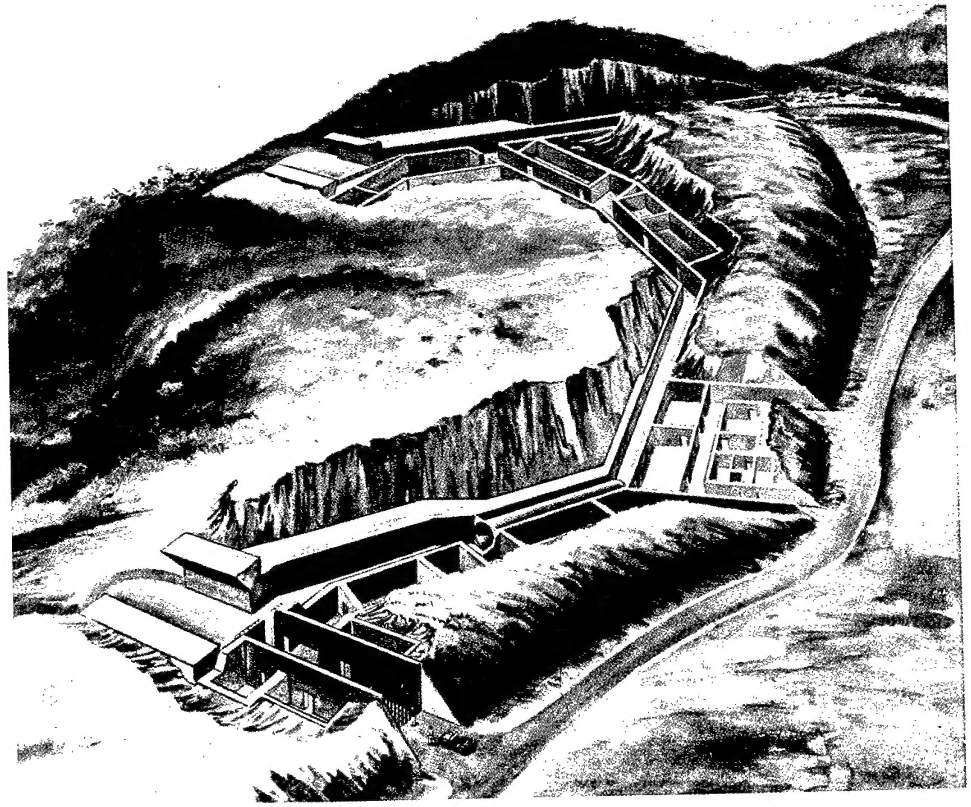
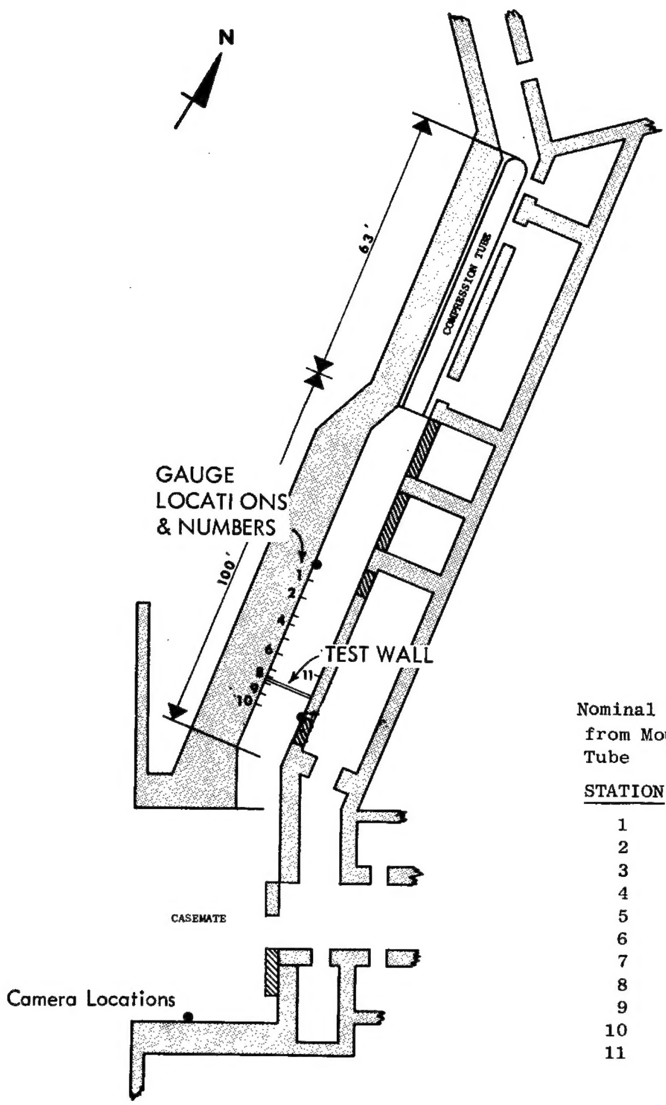


Fig. 1. Cutaway View of the URS Physical and Engineering Sciences Field Laboratory



Nominal Gauge Distances from Mouth of Compression Tube

STATION		ft
1	-	49
2	-	53
3	-	57
4	-	61
5	-	65
6	-	69
7	-	73
8	-	74
9	-	76
10	-	81
11	-	71.5

Fig. 2. Plan View of Shock Tunnel Facility

of the compression chamber portion of the tunnel. Unlike conventional compressed-gas shock tubes, it is not necessary to separate the compression chamber from the expansion chamber with a frangible diaphragm. The detonation of the primacord is sufficiently rapid (20,000 ft/second detonation velocity) that the pressure buildup is essentially quasi-static. The expansion of this high pressure gas into the remaining part of the tunnel (the expansion chamber) generates the desired shock.

Figure 3 illustrates the progress of a typical shock wave, its form and duration, for the open (no test wall) configuration. The facility is capable of generating an incident shock wave (overpressure) of about 12 psi, which is flat-top for about 40 msec, and a total positive phase duration of about 100 msec.

The support system is shown in Fig. 4 with an illustrative brick wall panel in a simple-plate configuration. The vertical plate girders are removable, providing a simple-beam support condition. The load cells provide data on load transmission to the support structure (tunnel walls) for both failing and non-failing wall panels.

THE TEST PROGRAM

The test program is divided into four basic parts which provide a functional separation of thought and deed:

- Static tests
- Loading study and tests
- Theoretical analyses
- Full-scale dynamic tests.

STATIC TEST PROGRAM

A static test program was conducted in conjunction with the shock tunnel dynamic tests to assure quality control in the construction of the test panels

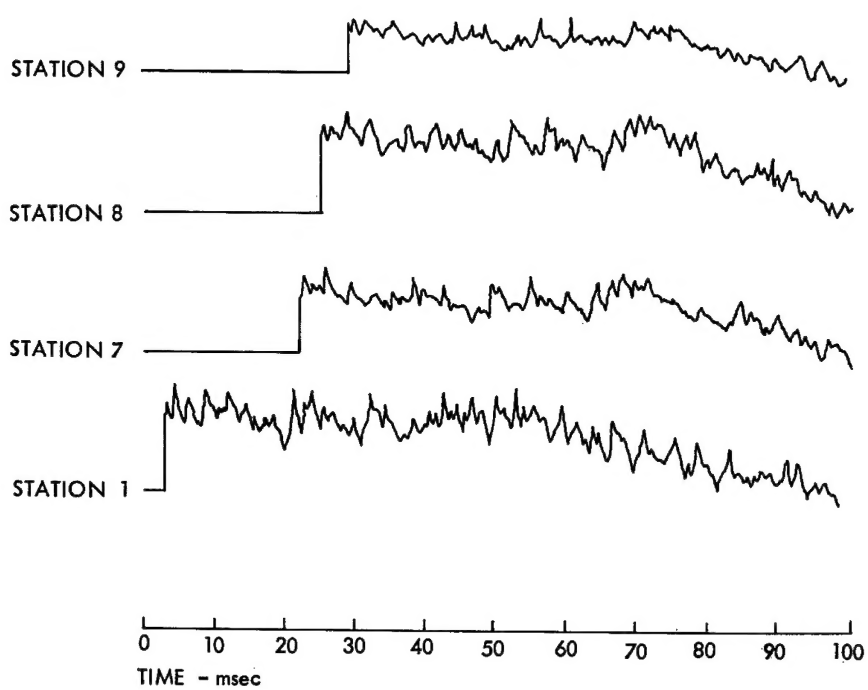


Fig. 3. Sample Data from Four-Strand Primacord Open-Tunnel Test

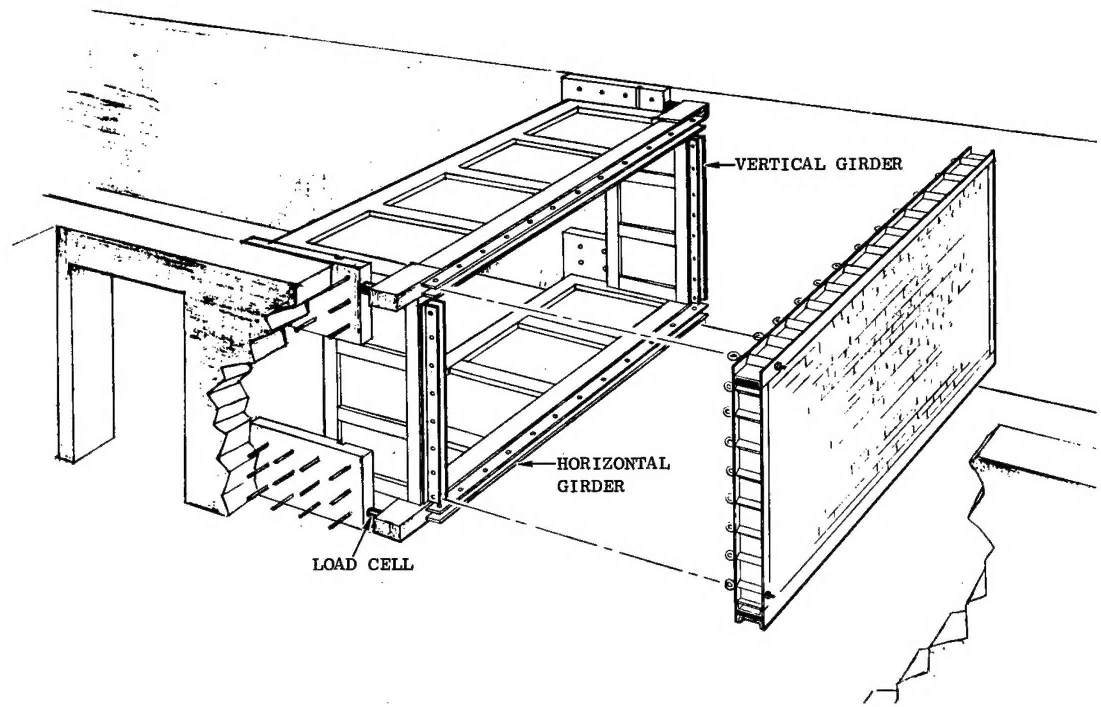


Fig. 4. Cutaway View of Shock Tunnel Showing Test Panel and Location of Horizontal and Vertical Plate Girders, Wall Blocks, and Load Cells

and to obtain estimates of the strengths of the panels at the time they were tested in the tunnel.

The tests performed were*

1. Brick and mortar beam tests for composite flexural behavior
2. Cylinder mortar tests for compressive strength
3. Cylinder mortar tests (splitting) for tensile strength
4. Brick flexure tests (flatwise) for tensile strength
5. Brick flexure tests (edgewise) for tensile strength and to obtain a measure of directional properties
6. Brick tests for compressive strength
7. Brick and mortar tests for compressive strength and composite modulus
8. Brick and mortar couplets for tensile-bond capacity
9. Brick and mortar tests for shear-bond capacity.

All tests were performed per ASTM standards, either sulphur or plaster of paris was used as the capping material.

Typical results are summarized below (in the same order as tests listed above):

1. Discussed in more detail below
2. Compressive strength (mortar)

   σ_c (mean) = 2656 psi

3. Tensile strength (mortar)

   σ_t (mean) = 522 psi

4. Flexural strength (brick, flatwise)

   σ_t (mean) = 835 psi

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* See Ref. 2 for more details.

5. Flexural strength (brick, edgewise)

σ_t (mean) = 703 psi

6. Compressive strength (brick)

σ_c (mean) = 3375 psi

7. Compressive strength modulic (brick-mortar composite)

σ_c (mean) = 4355 psi

E_c (mean) = 1.20×10^6 psi

8. Tensile strength (couplets)

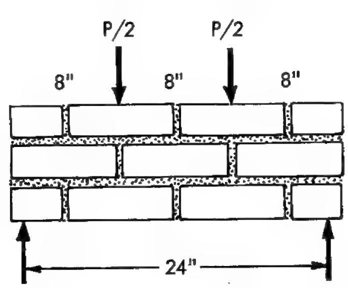
σ_t = 88 psi

9. Shear-bond strength (brick to mortar)

τ (mean) = 103 psi

The brick and mortar beam tests (No. 1) are probably the most revealing and meaningful of the static tests because of the manner of construction (like the walls) and the large size of the test specimens.

In these tests, brick beams approximately 26-1/2 in. long, 9 in. high, and 8-1/2 in. wide were loaded at the one-third points as shown below.



Since these tests are partly quality control tests, one beam is made for each wall and these in turn are made in relatively small batches. The results are summarized below.

Table 1

SUMMARY OF BRICK BEAM FLEXURAL TESTS TENSILE STRENGTH (σ_t)

WALL/BEAM NO.	BATCH NO.	σ_t (psi)
1	1	168
2		171
3		170
4	2	142
5		224
6		67.5
7		141
20	3	121
21		168
22		169
23	4	215
24		182
25		268
26		105
27	5	211
28		171
29		185
30		199
31	6	187
32		235
33		167
34		162

A plot of the foregoing data is shown in Fig. 5. All the data from all six batches are lumped together and treated as a single population.

From the foregoing static test data one can make an estimate of the test panel strength (Fig. 6). This estimate is after work done by Weibull,[3] Gumble[4] and modifications thereto by the author.[2] We observe that a design flexure stress of 20 psi, as recommended by Codes,[5] is not at all unreasonable. Also, these values are not unlike results published by other researchers[6,7] for static tests on full-scale walls (usually 4 ft by 8 ft by 8 in.).

For design, of course, we are interested in these lower strengths and probabilities of failure. However, for dynamic tests, we are interested in the higher strengths to assure a failure. Hence one may well expect flexural

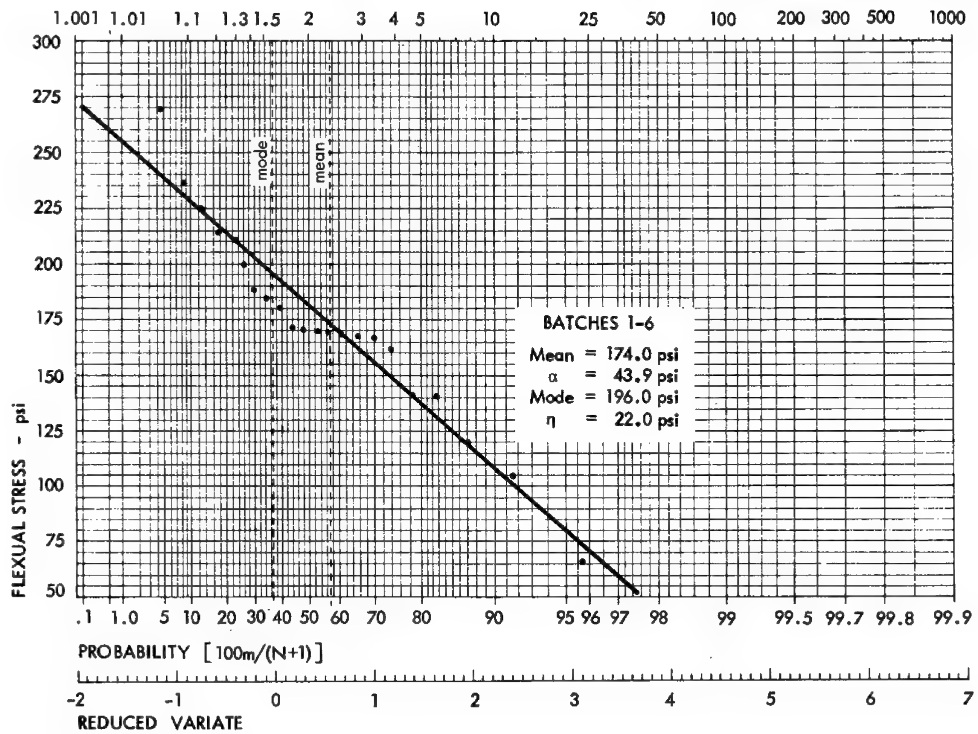


Fig. 5. Probability Plot of Flexural Stress (σ_t) from Brick Beam Tests

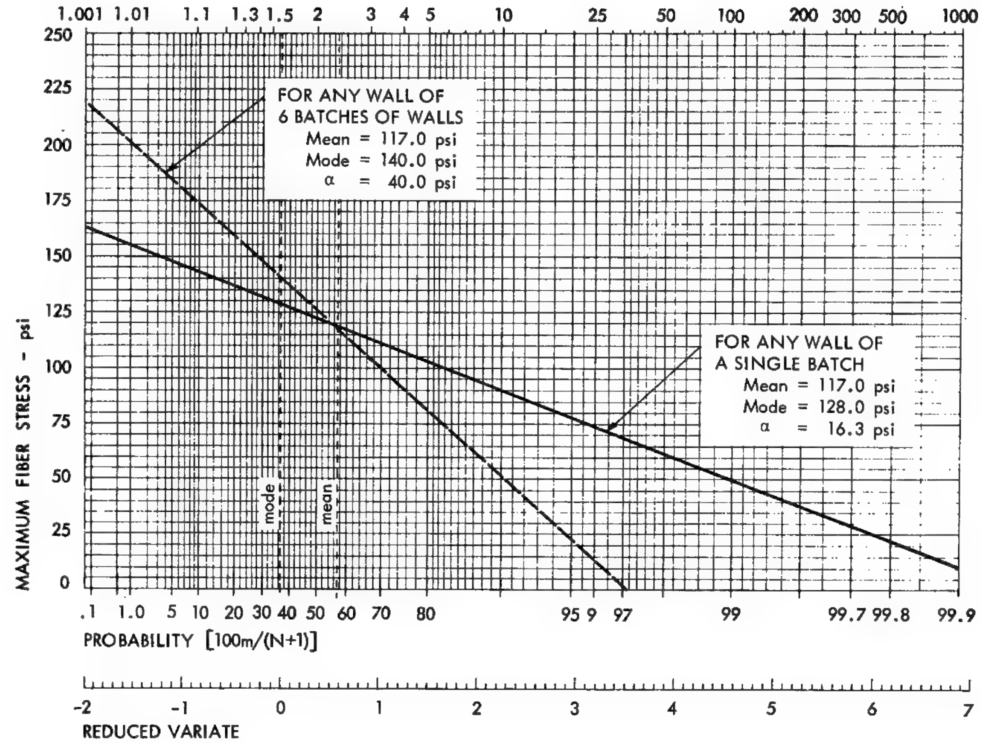


Fig. 6. Predicted Strength Distribution for 8' x 12' x 8" Brick Wall Panel

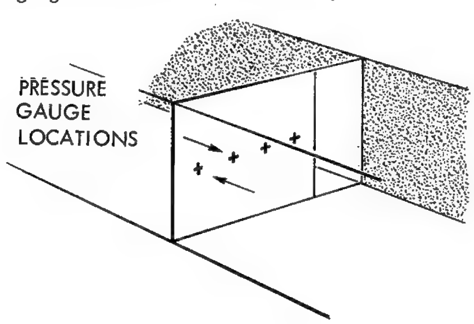
strengths in excess of 200 psi which should supply a loading adequate to fracture the wall.

LOADING STUDY

This portion of the effort is concentrated on developing a more accurate and complete description of the loading on structural elements. Obviously, the loading on either the fully closed or open tunnel is quite simple — merely a step load for 40 msec, then a decaying exponential. These modes have been used extensively in instrumentation evaluation and development. The more interesting loading cases are those on wall panels with openings, and on rooms. These more complex cases, coincidentally, are the cases for which the least is known; hence, where the research is concentrated.

The loading tests are carried out in the shock tunnel by installing instrumented modular non-failing walls (Fig. 7). The modular nature of the wall permits a wide variety of configurations to be tested.

The loading information, of course, is vital to the structural analysis of the test wall and support system, as well as analyses of other walls with similar configurations. For example, a doorway configuration such as shown below would have gauges located front and rear.



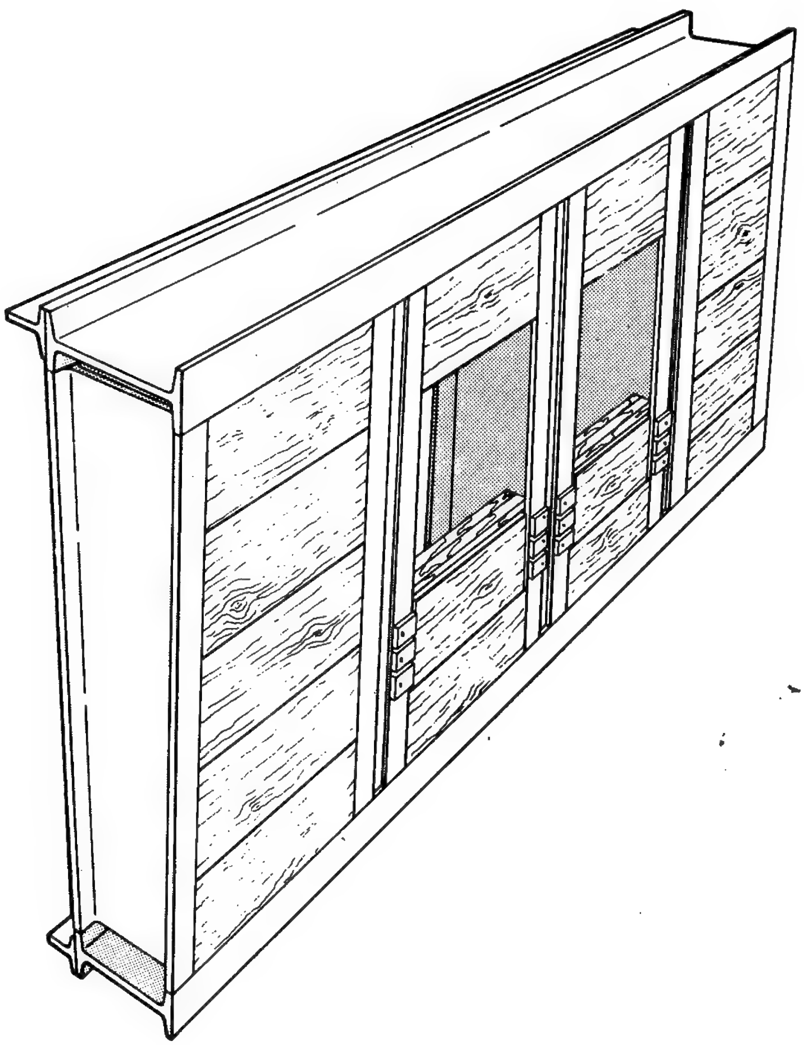


Fig. 7. Instrumented Non-Failing Wall

A series of tests is then conducted and pressure differentials at each location are established. Several replications at each pressure level are made and the results averaged. Figure 8 is a normalized net pressure vs time plot for four gauge locations. The normalizing is such that if the tunnel were completely closed the peak reflected pressure (p_f) would be 1.0 psi. Only 25 msec of data are shown as elastic failure occurs at an earlier time for brittle materials.

THEORETICAL (STRUCTURAL) ANALYSES

The foregoing pressure (loading) data are now available for structural analyses and response predictions. Initially these response analyses and predictions were made by hand, but as the structural and loading forms became more complex, the need for automated analyses is evident. Currently we are using a computer code (SAMIS)* developed by Philco-Ford Corporation, which is capable of dynamic analysis of finite element structural systems.

Figure 9 shows the basic element grid for our test system with line elements (2, 3, ·, ·, ·) representing the support plate girder (simple beam support condition) and the triangular finite elements, the wall. The shaded zone represents the doorway in our current example. From the static test program we obtain our material properties and loads from the loading study. The loads are shown as straight line approximations in Fig. 10 for the doorway and are applied in 2 ft strips to the wall panel.

Figures 11 through 15 illustrate some typical output of the SAMIS code. Figures 11 and 14 show that, although the major motion is downstream, a secondary motion is induced by the load varying across the face. This causes the

* Structural Analysis and Matrix Interpretive System.

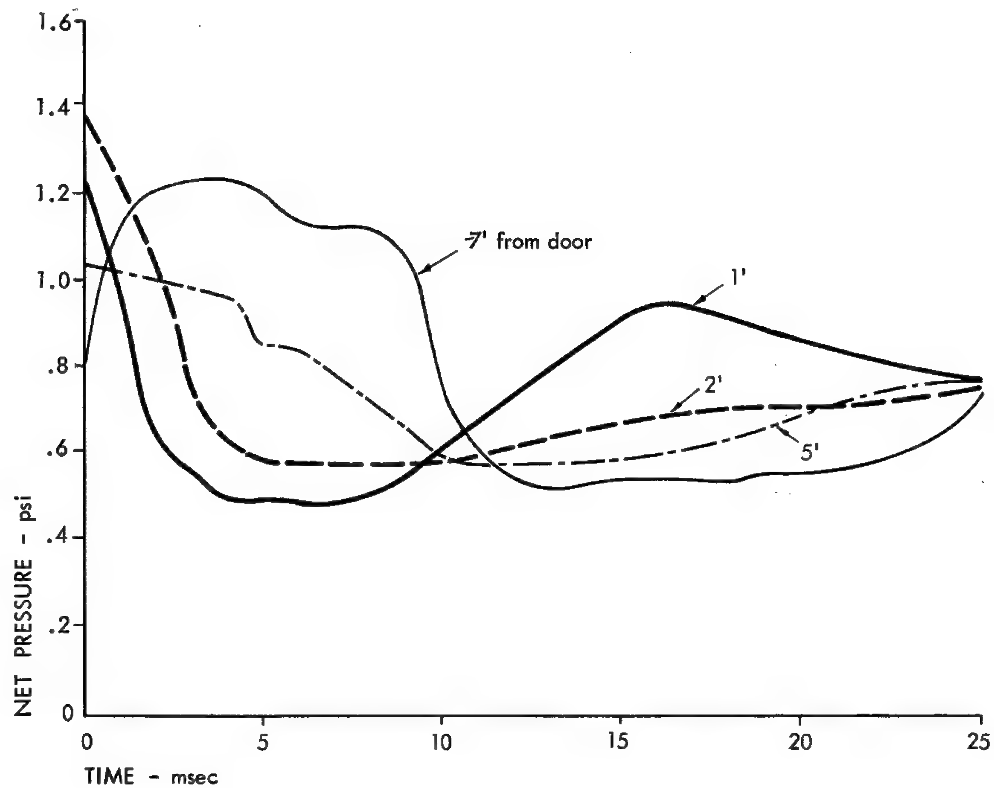


Fig. 8. Normalized Net Pressure vs Time

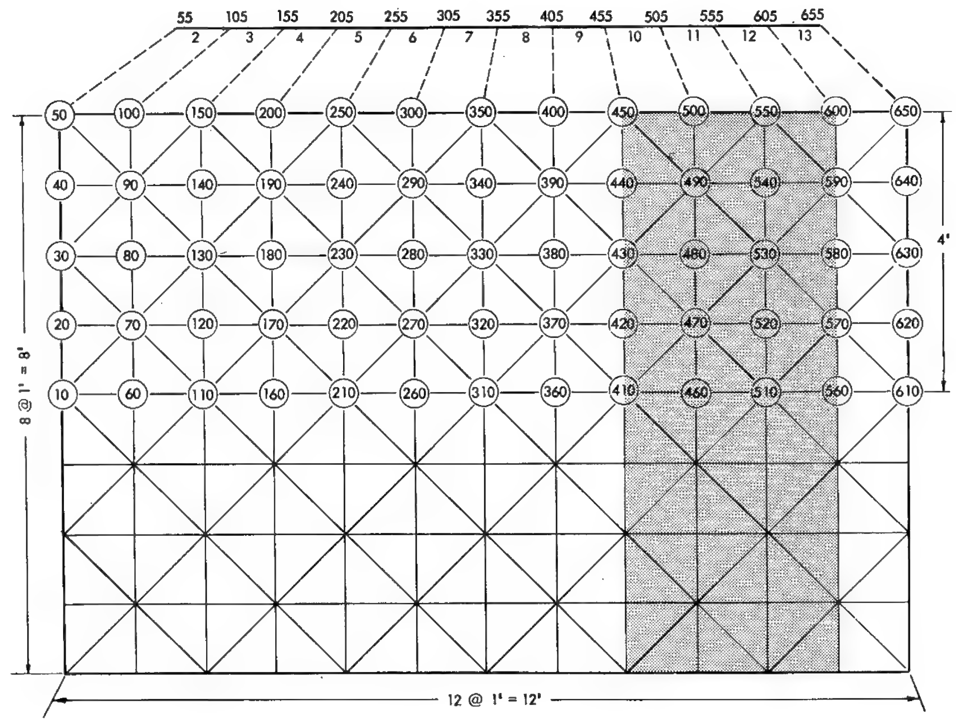


Fig. 9. Basic Grid and Line Elements

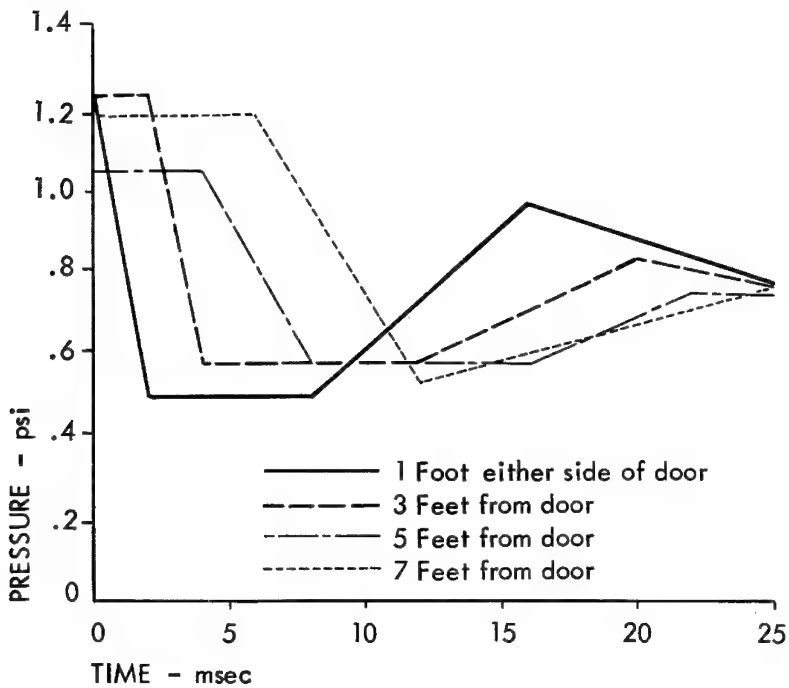


Fig. 10. Wall Loads

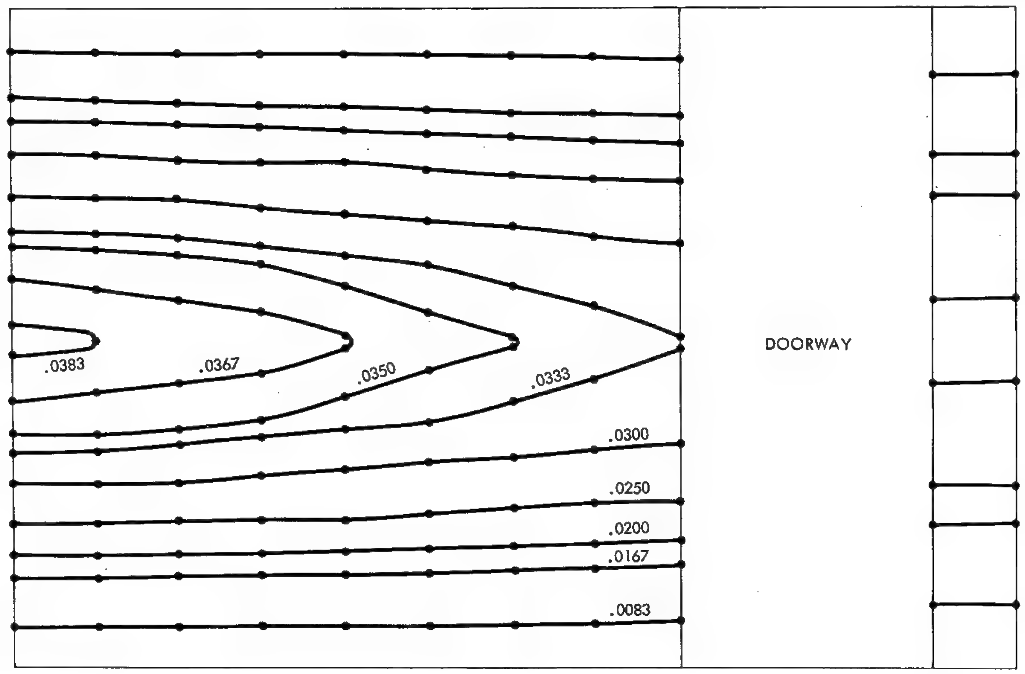


Fig. 11. Displacements at $t = 0.013$ sec for $p_f = 1.0$ psi

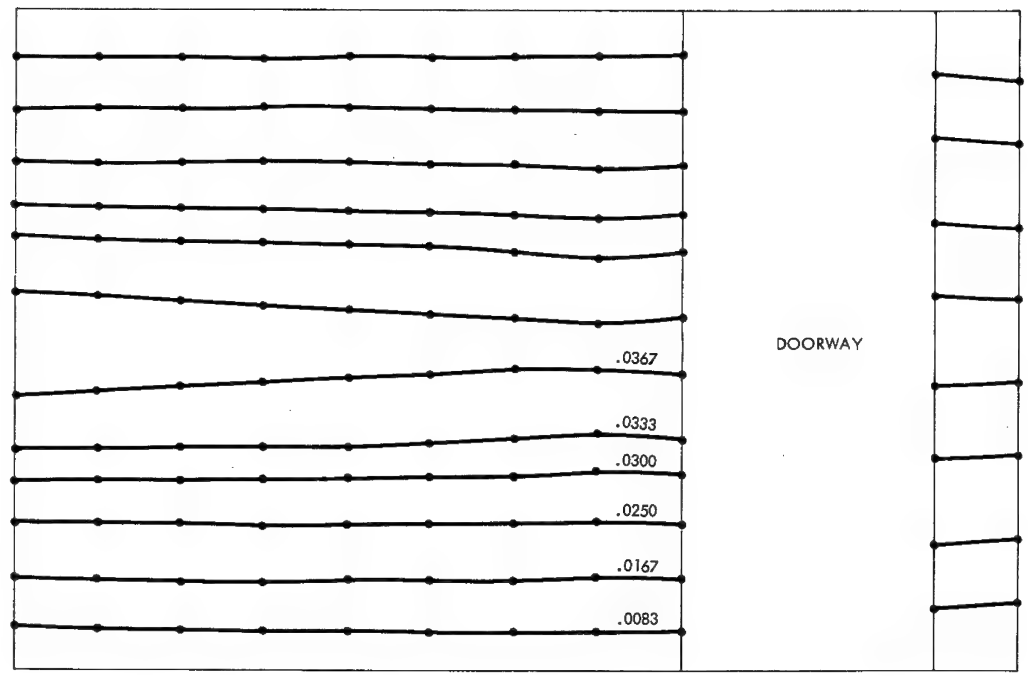


Fig. 12. Displacements at t = 0.014 sec for p_f = 1.0 psi

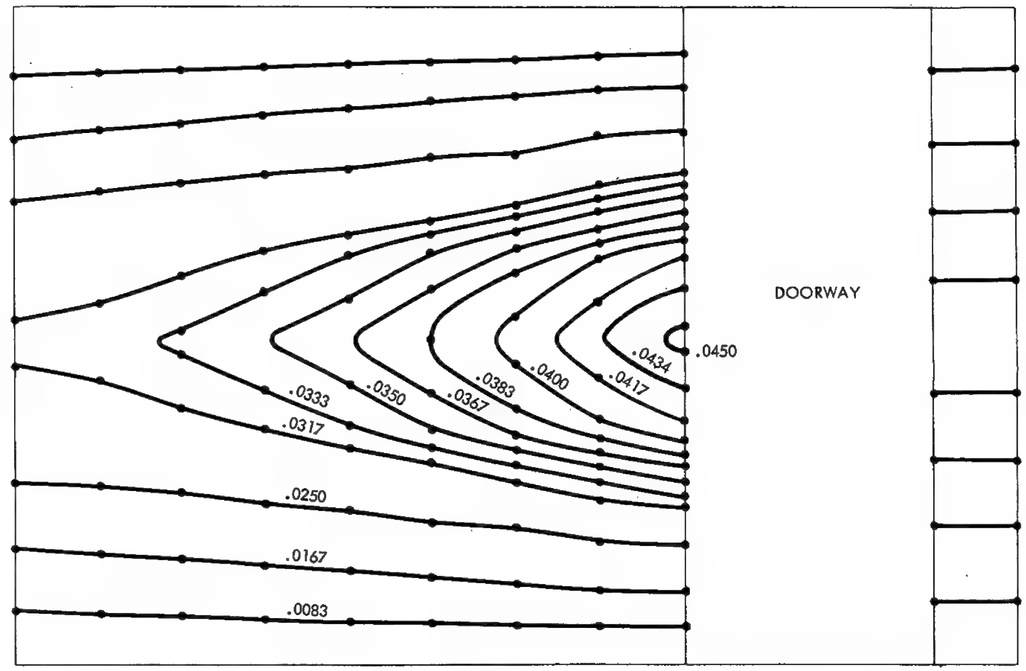


Fig. 13. Displacements at $t = 0.017$ sec for $p_f = 1.0$ psi

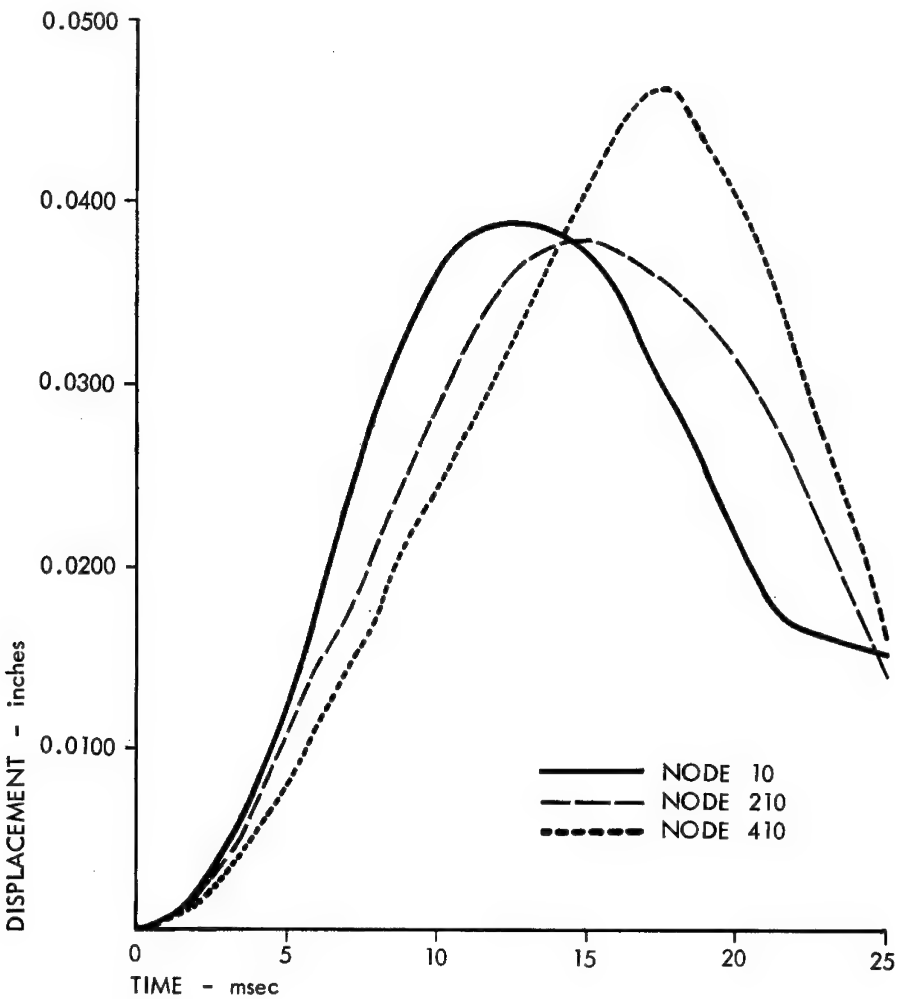


Fig. 14. Displacement vs Time

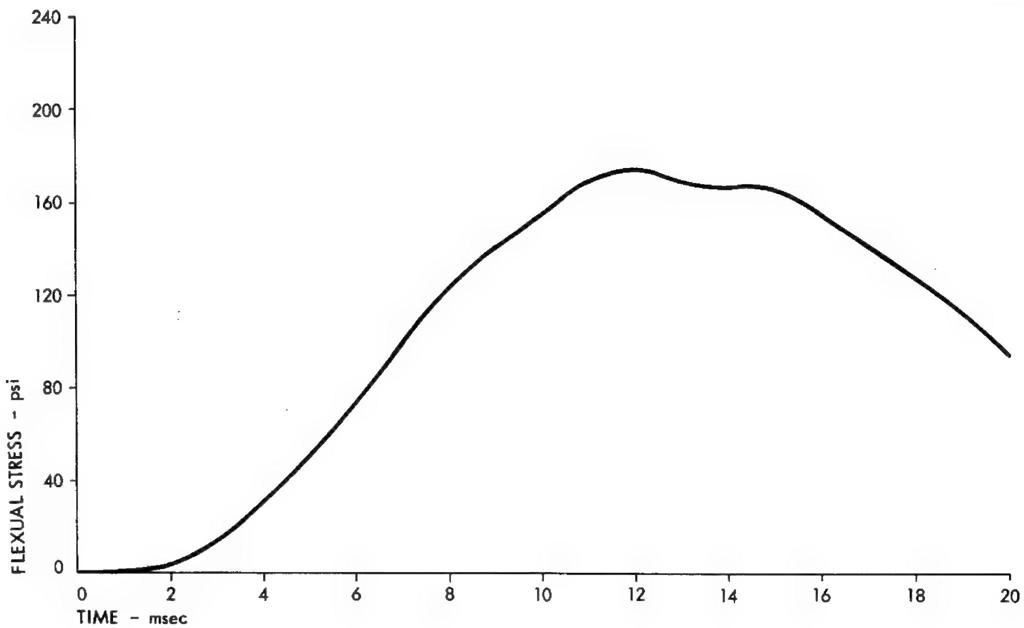


Fig. 15. Doorway Wall Panel Stress Area of Node 10

node 10 region to read maximum displacement and stress (Figs. 14 and 15) first at about 12 or 13 msec and node 410 later, at 18 msec.

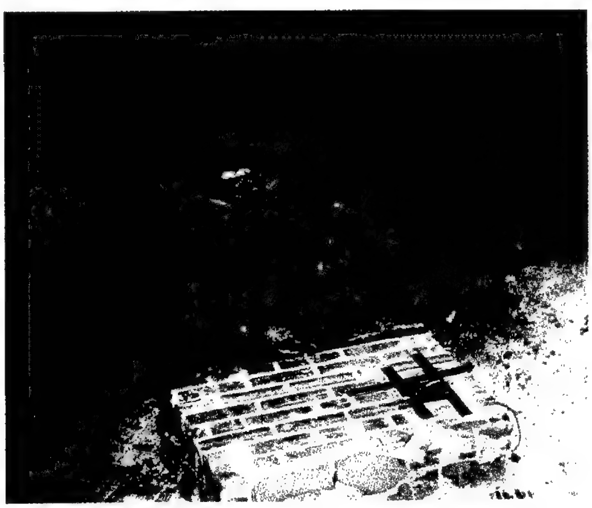
FULL-SCALE DYNAMIC TESTS

From the foregoing work we have the following input for a test of a full scale brick wall with a 3 ft doorway. That is an 8 ft by 8 ft by 8 in. brick wall panel with a 3 ft doorway. It is simply supported top and bottom, as shown in the following photograph.

From the static test program and the statistical failure theory one expects some probability of a wall having a flexural strength greater than 200 psi, see Fig. 6. From the dynamic analysis of the structural system we see

that a p_f of 1 psi provides a maximum stress of 170 psi; hence, we desire a slightly higher pressure to ensure failure on the first excursion and to eliminate possible low-level-fatigue (cracking).

During the course of the test program, it was found that a single strand of primacord supplies less than 1 psi overpressure and, further, that the waveform is inferior. Therefore, two strands of primacord were used, providing a lower bound p_f of 3 psi, which is much greater than the predicted need for fracturing the structure. In fact, from Fig. 14 and the static stress allowable, failure could be expected in 6 to 8 msec. The photograph below shows the brick wall after being subjected to 3.5 psi reflected overpressure.



WALL PANEL TESTS

A summary of the 32 wall-panel tests conducted to date is presented in Table 2. Included in this table are:

Table 2

SUMMARY OF PRELIMINARY LOADING AND RESPONSE DATA FOR WALL PANELS

Test No.	Loading Strands*	Loading P_f (psi)	Computed Applied Peak Load (lb)	Measured Peak at Load Cells (lb)	Dynamic Load Factor	Remarks**
		Brick Simple Beam Walls (8 in. thick)				
1	2	3	43,000	-	-	
2	2	3.5	50,000	-	-	
3	2	3.5	50,000	-	-	
5	2	3.6	51,000	92,000	1.8	
7	2	3.6	51,000	120,000	2.4	
21	2	3.4	47,000	82,000	1.7	
46	2	3.5	40,500	34,000	0.8	20% open door
4	4	10		-	-	
6	4	10.1	142,000	42,000	0.3	
20	4	10.3	145,000	195,000	1.4	
22	4	7.6	107,000	126,000	1.2	
44	4	9.5	110,000	116,000	1.1	20% open door
		Brick Simple Beam Walls (12 in. thick)				
50	2	4.0	58,000	-	-	
51	2	4.3	62,000	-	-	
		Sheetrock Simple Beam Walls (4 in. thick)				
8	2	3.3	47,000	42,000	0.9	
10	2	2.4	34,000	46,000	1.4	
9	4	7.0	99,000	70,000	0.7	

* Of primacord.

** All panels collapsed unless otherwise noted.

Table 2, cont.

Test No.	Loading Strands	Loading P_f (psi)	Computed Applied Peak Load (lb)	Measured Peak at Load Cells (lb)	Dynamic Load Factor	Remarks
			Brick Simple Plate Walls (8 in. thick)			
24a	2	3.2	46,000	-		Wall did not collapse
24b	2	3.0	43,000	43,000	1.0	Second loading
25	2	3.5	51,000	74,000	1.5	Wall did not collapse
29a	2	3.8	55,000	86,000	1.6	Wall did not collapse
29b	2	4.2	61,000	90,000	1.5	Second loading
28	2-1/2	4.0	58,000	62,000	1.1	
23	4	10.9	157,000	235,000	1.5	
32	4	9.3	134,000	166,000	1.2	
33	4	9.3	134,000	189,000	1.4	
26	6	19.3	259,000	-	-	
30	6	16.6	240,000	235,000	1.0	
31	6	15.0	216,000	304,000	1.4	
			Concrete Simple Beam Walls (8 in. thick)			
36	2	3.5	50,000	76,000	1.5	

- The input loading - the number of strands of primacord and the measured peak reflected overpressure
- The computed applied peak load - the product of the wall area parallel to the shock front and the peak reflected overpressure
- Total load - as measured by the four load cells located as shown in Fig. 4
- The dynamic load factor - the ratio of net load, as measured by the load cells, to the computed applied peak load.

These tests were conducted over a period ranging from early 1967 to August 1969.

The fundamental philosophy used throughout this entire long-range program has been to conduct tests primarily to furnish specific data inputs to the theoretical efforts which have been proceeding in parallel with the experimental program. For this reason, the data requirements for these tests have changed markedly throughout the program and will continue to do so. In the early tests, the data obtained consisted of peak incident and reflected overpressure monitored by gauges in the tunnel wall and the total reaction as measured by the load cells.

As the program evolved, the demands of the various theoretical efforts have changed the data requirements to include:

- the natural period of the wall panel as installed in the tunnel
- the shock wave arrival time on the front face of the wall panel
- the time of first crack
- the initial crack pattern on both the front and back faces of the wall panel
- strain gauge measurements.

A great deal of the current effort is concentrating on the post-fracture response of the wall which will, in all probability, change the data demands again.

CONCLUSIONS

Basic data relating to the response, loading and debris characteristics of wall panels are being provided by this on-going test program. The program to date has been concentrating on brittle (non-reinforced brick) wall panels with various beam and plate support conditions, with and without openings. The panel behavior is basically classical in nature until fracture, that follows the laws of classical mechanics and statistics. Hence, post-fracture behavior has received a good deal of attention in the program as the entire failure process is of interest.

NOMENCLATURE

σ_c	compressive strength (psi)	σ_t	tensile strength (psi)
τ	shear strength (psi)	E_c	modulus of elasticity (psi)
p_i	incident pressure (psi)	p_f	reflected pressure (psi)

REFERENCES

1. Willoughby, A. B., C. Wilton, and B. Gabrielsen, Development and Evaluation of a Shock Tunnel Facility for Conducting Full-Scale Tests of Loading, Response, and Debris Characteristics of Structural Elements, URS 680-2, Subcontract Nos. 11229(6300A-320) and 11618(6300A-250), URS Research Co., Dec. 1967.

2. Willoughby, A. B., C. Wilton, B. L. Gabrielsen, and J. V. Zaccor, A Study of Loading, Structural Response, and Debris Characteristics of Wall Panels, URS 680-5, URS Research Co., Contract No. 11618(6300A-250), July 1969.

3. Weibull, W., "A Statistical Theory of the Strength of Materials," Handling No. 151, Engineering Academy, General Lithographic Library, Stockholm, Sweden, 1939.

4. Gumbel, E. J., "Theory of Extremes," Columbia University Press, 1958.

5. "Uniform Building Code," 1961 Edition, Volume 1, International Conference of Building Officials.

6. Monk, C. B., Resistance of Structural Clay Masonry to Dynamic Forces, Research Report No. 7 of the Structural Clay Products Research Foundation, Geneva, Illinois, Nov. 1958.

7. Fishburn, C. C., Effect of Mortar Properties on Strength of Masonry, National Bureau of Standards Monograph 36, Nov. 20, 1961.